DESIGN, CONSTRUCTION, AND OPERATION OF STREAMFLOW-MEASURING FACILITIES IN THE LITTLE RIVER WATERSHED, GEORGIA

ARS-S-148

October 1976

AGRICULTURAL RESEARCH SERVICE • U.S. DEPARTMENT OF AGRICULTURE

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Agricultural Research Service
UNITED STATES DEPARTMENT OF AGRICULTURE
in cooperation with
University of Georgia
and
Middle South Georgia Soil Conservation District

DESIGN, CONSTRUCTION, AND OPERATION OF STREAMFLOW-MEASURING FACILITIES IN THE LITTLE RIVER WATERSHED, GEORGIA

By Paul Yates[1]

ABSTRACT

Instrumentation necessary for the measurement of streamflow in a Coastal Plain watershed is discussed. Flow in low-gradient streams of the area must be confined at highway bridges or box culverts to be measured. Construction of measuring weirs creates two problems: either weirs will be submerged for much of the flow, or backwater ponding will be severe. Model studies were conducted to develop optimum design of flow-measuring facilities. Construction and operation of facilities are discussed and illustrated. An evaluation of the operation and performance is given. General comments and recommendations for future construction are presented.
KEYWORDS: Coastal Plain hydrology, hydrology, stream gaging, stream-gaging facilities, streamflow measurement, V-notch weirs.

INTRODUCTION

Background

Although research in the hydrology of agricultural watersheds has been underway in many areas since the 1930's, the Coastal Plain area of the Southeastern United States has received little attention. Because the area is blessed with heavy rainfall (approximately 46 in annually) (*10*),[2] little thought was given to the need for hydrologic research here. Any attention the area might have received was moderated by the difficulty and expense of confining flow into satisfactory measuring sections.

Watersheds in this region are characteristically marked by broad, flat, alluvial flood plains with low-gradient, poorly defined channels. Above the lowest flows, channels become nonexistent, with flows spreading across the wide, wooded flood plains. The lack of channels is illustrated in figure 1, which shows the outlet of the proposed Little River watershed gaging site A. This point on Little River at U.S. Highway 82, just west of Tifton, Ga., must accommodate flow from 145 mi^2 (fig. 2). For economic reasons, flow-measuring installations must be placed at highway bridges or culverts, the only points where streamflow can be properly and practically confined.

The Southeast Watershed Research Center, later named the Southeast Watershed Laboratory (SEWL), authorized by Congress in October 1965, was assigned to identify and characterize the elements that control the flow and quality of water from agricultural watersheds in the Southeast, with major emphasis on the Coastal Plain Land Resource Area. In January 1966, SEWL initiated a research program to study the hydrologic behavior of agricultural watersheds in the Coastal Plain. The Little River watershed (LRW) in Tift, Turner, and Worth Counties, Ga., was chosen as the major research area.

A comprehensive watershed hydrology study requires the measurement of all inputs and

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[2] Italic numbers in parentheses refer to items in "Literature Cited" at the end of this publication.

FIGURE 1.—Outlet of Little River at proposed gaging site A.

outputs, as well as the quantification of the watershed characteristics affecting hydrologic behavior. Instrumentation in the LRW included networks of precipitation gages, streamflow stage recorders, and ground-water-well stage recorders. Such factors as soils, land use, topography, geology, and geomorphology were also evaluated. The research program, watersheds, and instrumentation are discussed in references *6* and *13*. This report primarily provides details of the construction, instrumentation, and operation of the streamflow-measuring facilities in LRW.

Little River Watershed

The upper Little River watershed was selected as the major research area because it is typical of the Coastal Plain farming areas, and because it has a stream system that could be instrumented to obtain satisfactory flow measurements at a reasonable cost. Also, the contribution of surface water to deep seepage was thought to be relatively insignificant. The originally selected study area of 145 mi^2 above proposed gaging site A was later reduced to the 126.4-mi^2 area above gaging site B (fig. 2). As shown in the figure, numerous initial sites considered for instrumentation were not developed: some were found to be hydrologically undesirable, and others were found to be unnecessary or too expensive to instrument. For example, station A with a valley width of approximately 1,800 ft drained through three bridge openings with a combined width of 562 ft. Although the drainage area of site A would have been 19 mi^2 larger than site B, the additional research information that would have been gained could not justify the large additional expense.

DESIGN CRITERIA

Streambed elevations in LRW drop about 3 to 5 ft/mi, a slope of less than 0.1%. Design of streamflow-measuring structures in these low-gradient streams presents two problems: (1) the free outfall required by most flow-measuring devices is difficult to obtain, and (2) raising the control only 2 ft may cause ponding as far as 0.5 mi upstream. A compromise between backwater ponding effects and control submergence was necessary in the design and construction of the control.

The broad-crested Virginia V-notch weir was selected as the most desirable flow-measuring structure for use in LRW. This weir is an economical and efficient flow-meauring installation when properly designed and located with respect to highway culverts (*1, 4*). The design provides accuracy over a wide range of flow rates, is sensitive at low flow rates, and is relatively inexpensive to install and maintain. However, it is usually rated for free-outfall conditions. In the low-gradient streams of the Coastal Plain, free discharge may be obtained only at the lower flow rates. Because weir crests must be installed relatively close to channel beds, thereby reducing undesirable backwater effects, weirs of permissible heights may be subjected to submerged flow during much of the discharge period, and definitely during all periods of moderate and high flows. Model studies were made to test and calibrate the Virginia V-notch weir under simulated LRW conditions.

Model Studies

Model studies were conducted at the ARS Water Conservation Laboratory in Stillwater, Okla., and in the University of Georgia Agricultural Engineering Department Soil and Water Laboratory in Athens, Ga. The objectives of these model studies were (1) to develop design criteria that would maximize accuracy and efficiency of weirs, (2) to minimize undesirable factors, (3) to determine the effect of the weirs on the capacity and stability of highway bridges and culverts, and (4) to determine the head-discharge ratings of the structures (*2, 5, 11*).

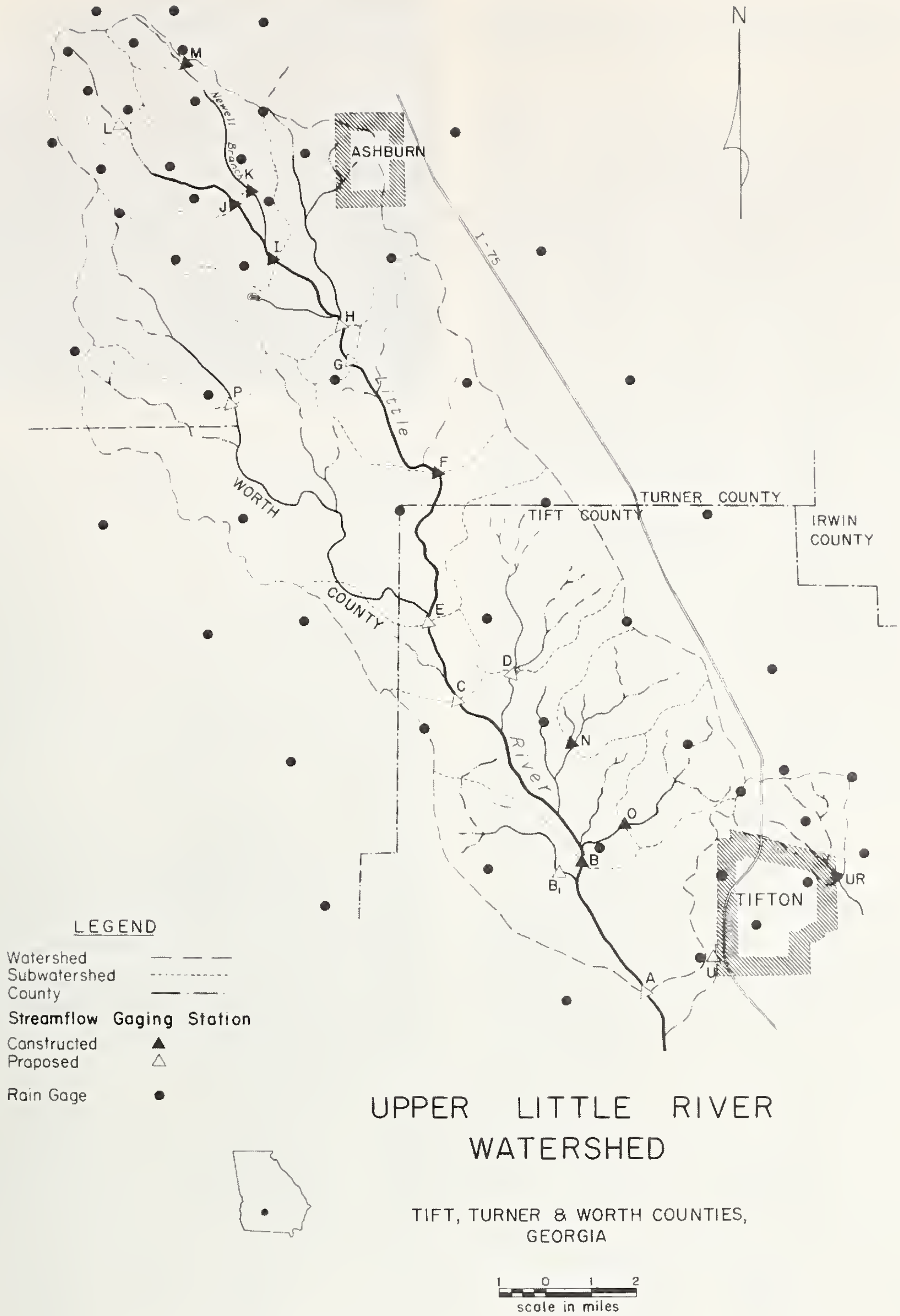

FIGURE 2.—Little River, Ga., watershed map.

The model studies produced appropriate criteria without modification of the basic weir design. However, modeling provided data for a modified design of the downstream apron, or stilling basin, to minimize scour downstream from the apron. As originally designed, the aprons allowed scour to undesirable depths. Because of the likelihood of scour underneath the apron itself, which would place the entire structure in danger of an eventual collapse, the apron design was modified by the addition of baffle blocks and a flip sill. Also, the down-

stream apron length was reduced from 25 to 10 ft, appreciably reducing construction costs (*2*). The model of station B is shown in figure 3.

The model studies also indicated that submergence effects begin when the ratio of the downstream stage to the upstream stage (submergence ratio) equals about 0.60. Free fall with nappe aeration is restricted at this stage, and flow depth at the weir crest becomes greater than the critical depth. Weir stage-discharge relations, as well as submergence effects on discharge, are discussed in detail in references *2* and *4*.

No evidence was found to indicate that weirs have significant effects on either the capacity or the stability of highway bridges and culverts.

Weir Design for Watershed Sites

Under submerged flow conditions, a change in the downstream flow depth causes a change in the upstream flow depth for a given constant discharge rate. Weir ratings based entirely on upstream levels may no longer be applicable: both upstream and downstream head readings are necessary. Therefore, streamflow measuring installations in LRW are designed to record both headwater and tailwater surface elevations simultaneously, with the exception of station M, which is discussed in a later section.

The measuring facilities were designed and constructed to contain all expected flow within the V-notch portion of the weir 90 to 95 percent of the time. This amount represents ap-

FIGURE 3.—Model of gaging site B.

proximately 65 to 70 percent of the total flow volume, based on anticipated 25-year flow rates.

The determination of these anticipated flow rates was achieved by analyzing the limited available U.S. Geological Survey flow data for nearby Coastal Plain streams and by developing flood-frequency curves. With the developed frequency flow rates, coefficient (C) values were calculated for the Cypress Creek equation (*7*)

$$Q=CA^{0.833}, \qquad (1)$$

where Q=discharge (ft^3/s), C=a coefficient, and A=drainage area (mi^2).

The C values determined for the various streams were plotted against the return period. With these plots, a C value of 108 was determined for the anticipated 25-year flow rate in

TABLE 1.—*Gaging-site statistics*

Site	Drainage area (mi^2)	Date flow records began	Anticipated 25-year flow rate (ft^3/s)	Maximum flow rate, 90% of time[1] (ft^3/s)	Maximum flow rate, 90% of total volume[2] (ft^3/s)
B[3]	126.40	Oct. 14, 1971	6,760	303	1,249
F[3]	45.60	Nov. 29, 1968	2,891	110	450
I[3]	20.26	Dec. 8, 1967	1,471	48.6	200
J[4]	8.81	Dec. 1, 1967	735	21.1	87
K[4]	6.66	Dec. 6, 1967	582	16	65.8
M[4]	1.20	Dec. 6, 1967	140	2.88	11.8
N[4]	6.15	Oct. 3, 1970	545	14.8	60.8
O[4]	5.72	Nov. 29, 1968	513	13.7	56.5
UR[4]	3.17	Aug. 7, 1969	314	7.61	31.3

[1] Flow rates will be equal to or less than these values 90% of the time.
[2] 90% of total flow volume will occur at or less than these flow rates.
[3] Highway bridge.
[4] Concrete box culvert.

Little River. Allowing for a safety factor of 10 percent, the C value was increased to 120. Calculated probable 25-year flow rates for Little River gaging sites, along with other pertinent data, are shown in table 1.

The weirs were designed for accurate measurements, ranging from zero discharge to the 25-year flood, but ample freeboard was allowed to accommodate the 100-year flood without damage to the structure.

Site Selection

Sites were selected on the basis of drainage-area size, relationship to other sites, and suitability for instrumentation. An effort was made to instrument sites draining areas of various sizes. As shown in table 1, drainage areas chosen range from 1.2 to 126.4 mi^2. Also desired was an arrangement whereby the sites could be studied in parallel and in tandem. As shown in figure 2, sites J and K are parallel, as are sites N and O. Sites J, I, F, and B, all on the main channel, allow for study of tandem sites increasing in drainage area size in a downstream order. One site, UR, an adjacent watershed that is being changed from agricultural to urban uses, was instrumented to study the effects of urbanization on watershed hydrology.

For all sites selected, easements were obtained from the appropriate highway departments: the Georgia State Department of Highways; Tift County, Ga., Commissioners; and the Turner County, Ga., Commissioners. Each of these agencies plan to utilize information gained to aid in their design of bridge and culvert openings on Coastal Plain roadways. Because no costs were involved in the easements, ARS proceeded with construction and instrumentation soon after submitting detailed plans and specifications for approval by the respective governing authorities. ARS agreed to abide by all highway department safety regulations during construction, to maintain all facilities in a safe and neat condition, and to be responsible for removing installations upon conclusion of the research.

At the sites where the construction extended beyond the highway right-of-way onto private property, long-term leases were made with the landowners. Since all sites are in an area of the flood plain that is largely swampy and unused, few problems were encountered in obtaining the necessary leases. Each landowner was paid a small annual rental fee, and cooperation of the landowners has been excellent.

Weirs were located 25 ft downstream from highway bridges. At culvert sites, weirs were located between the outer ends of the upstream culvert wingwalls, a distance usually of approximately 10 ft upstream from the culvert.

CONSTRUCTION OF STREAM-FLOW-MEASURING FACILITIES

Contracts

After serious consideration and analysis of various types of contracts, a cost-plus contract was selected. The contractor was reimbursed for all expenses plus a cost percentage as his salary and profit. The contract allowed for the flexibility needed to make any unexpected modifications that became necessary. Such changes could then be made immediately with no time loss while additional cost negotiations were undertaken. Modifications were necessary at times since this was a new type of construction for both the contractor and ARS. Also, since the work was within stream channels, flow conditions could not be predicted several weeks in advance. When high flows occurred, construction procedures and schedules were greatly modified.

Onsite administration of the contract was charged to the Government construction superintendent who coordinated all work with the contractor's foreman. Each work phase was approved by the superintendent before the next could begin. The contractor's log was checked daily for any possible discrepancy, the disagreements being resolved each workday. The ARS construction superintendent controlled all phases of construction, assuring the quality of materials and workmanship, as well as maintaining an acceptable construction schedule.

ARS had purchased by competitive bids most materials required in large amounts, including such items as sheet-steel piling, H-beams, treated timber piles, lumber, concrete, and reinforcing bars. Other materials and labor were furnished by the contractor, as were most tools and equipment at specified rental rates.

Installation Scheduling

Since work was to be within and immediately adjacent to stream channels, construction was scheduled for the dry season. Site preparation

usually began in early July or as soon thereafter as weather permitted, allowing work to be done within the channel during the dry months beginning in September. Differences in stream conditions between the dry and wet seasons are illustrated in figure 4 and 5. In figure 4, note the dry valley conditions. Not seen in this view is a small amount of confined flow that can easily be bypassed. Figure 5 illustrates the conditions during a moderate discharge: water is spread across the entire valley, prohibiting construction without expensive dewatering practices.

Site Preparation

A minimum amount of site clearing was required since sites were largely within the normally clear highway right-of-way. At some sites fences and utility lines had to be relocated before construction could begin. Site preparation consisted primarily of land smoothing and filling, stockpiling materials, and providing road-safety precautions. The site of the weir and stilling apron was filled to the desired elevation, compacted, and leveled. Generally, sufficient soil was available at the site to prepare for construction. Coffer dams were constructed upstream when necessary to eliminate flow through the construction area. The flow was pumped around the work area and allowed to return to the channel system farther downstream. Figure 6 shows a site prepared for construction.

Approximate steel-piling lengths were determined by borings made across the length of the proposed weir site to determine depths through the valley alluvium to the more impervious subsurface material. Steel was then precut, increasing installation efficiency, decreasing waste, and reducing costs. Interlocking steel piling extended into the impervious material, serving as a cutoff wall to decrease seepage and the possibility of piping.

Construction

Components to be constructed included guidewalls, wingwalls, weirs placed on cutoff walls, stilling aprons, stilling-well-recorder shelters, intake assemblies, and footbridges.

Guidewalls and wingwalls

The flow of water must be confined to a suitable section to insure reasonable accuracy

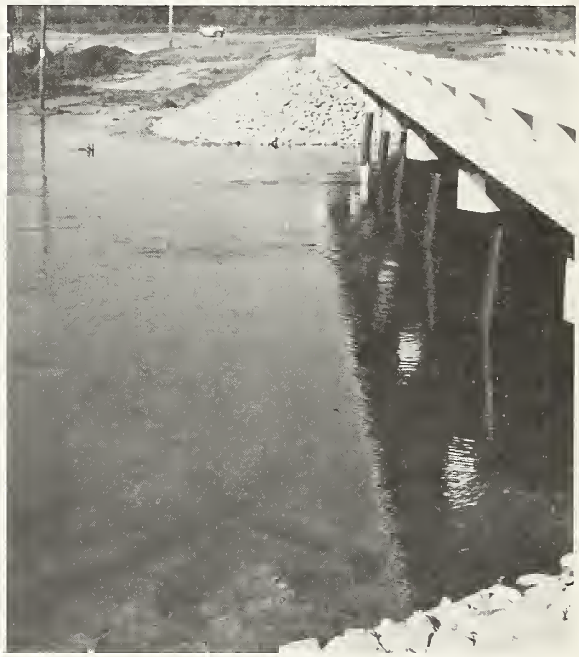

FIGURE 5.—Little River gaging site B (wet season).

FIGURE 4.—Little River gaging site B (dry season).

FIGURE 6.—Gaging site ready for construction.

FIGURE 7.—Threading sheet-steel piling utilizing H-pile wales and struts as guides.

FIGURE 8.—Outside guide-wall bracing with riprap downstream of apron.

in the measurement of flow rates. At highway bridges, such restriction is attained with guidewalls extending downstream from the bridge for the necessary distance past the weir. The guidewalls act as walls of a flume, constricting the flow so it passes over the weir. A T-section, U.S. Steel WT12B, was installed in each guidewall at the point where the weir was to be located to tie together the guidewalls and weir cutoff walls, eliminating leakage around the ends of the weirs.

The guidewalls were constructed with U.S. Steel MP–115 interlocking sheet-steel piling, which provides a cutoff wall that is virtually watertight when properly installed. (Specifications of this material are given in reference *9*). All sheet-steel piling was sanded and painted upon receipt, then stockpiled at the construction site. A good grade red lead paint inhibited rust and facilitated installation.

Wales and struts of 8- by 8-in H-piles were installed to guide steel piles into proper position (fig. 7). Pile interlocks were then threaded in place. After all piles were in position, they were driven to desired depths, and the tops were cut to correct elevations. After all piling was in place, the guidewalls were braced from the outside to insure wall stability during high flow rates (fig. 8).

Wingwalls parallel to the roadway are connected to the guidewalls by means of a 90°-angle piling section (fig. 9). The wingwalls extend for 8 to 10 ft to prevent high flows from passing around the guidewalls, preventing not only flow loss from the measuring section but also erosion of the steep roadbanks.

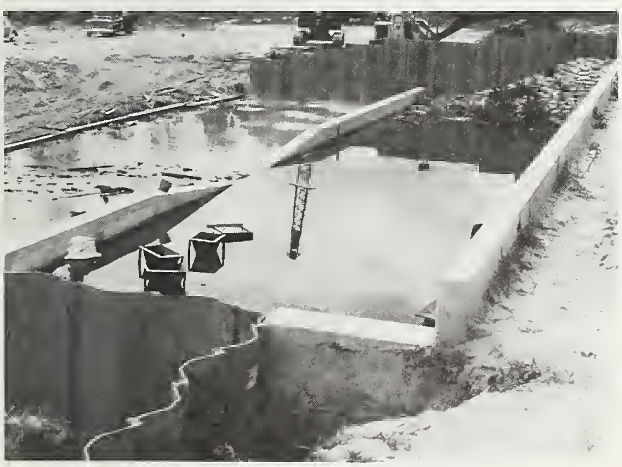

FIGURE 9.—Steel-piling wingwalls parallel to roadway.

FIGURE 10.—Box-culvert wingwalls, original sloping crests.

Steel-piling guidewalls were not required at locations utilizing concrete-box culverts as measuring sites. Weirs were placed within the outer end of the existing upstream concrete wingwalls. As originally constructed, these wingwalls all sloped downward from the culvert to the outer ends (fig. 10). To confine the streamflow, it was necessary to modify these walls by extending the sloping section upward to a horizontal level (fig. 11).

Weirs

Weirs at bridge sites were placed approximately 25 ft downstream from the bridge. As explained in the preceding section, weirs were placed upstream of box culverts. Construction details were identical for both placements: each weir was built atop a steel wall that aids as a support for the concrete weir as well as an impediment to subsurface flow. The curtain wall was constructed of the same material and in the same manner as the guidewalls. After steel piling had been driven into place, it was cut off to the desired grade (fig. 12). Strength and support of the weir was provided by 8-in H-beams driven at 12- to 15-ft intervals across the wall and welded to the sheet piling. H-beams were driven approximately 5 ft deeper than sheet piling.

Prior to the installation of forms for construction of the concrete weir cap, steel reinforcing bars, three-quarter inch in diameter, were assembled as shown in figure 13. Horizontal reinforcing bars were spaced 5 in on center with tie bars and stirrups spaced 19⅝ in on center. This spacing, which was found to be adequate, corresponds to the width of each individual section of interlocking piling. Tie bars were welded to sheet piling at the centerline of each piling section.

Forms for the sides of the concrete weir were fabricated in advance to reduce field time and to insure accuracy and uniformity of weir construction. Special sheet-metal forms (fig. 14) constructed in a machine shop shaped the crest in accordance with design specifications given in ARS Agriculture Handbook 224 (*8*). Special

FIGURE 11.—Box-culvert wingwalls, modified horizontal crests.

FIGURE 12.—Sheet-steel-piling cutoff wall cut to desired grade.

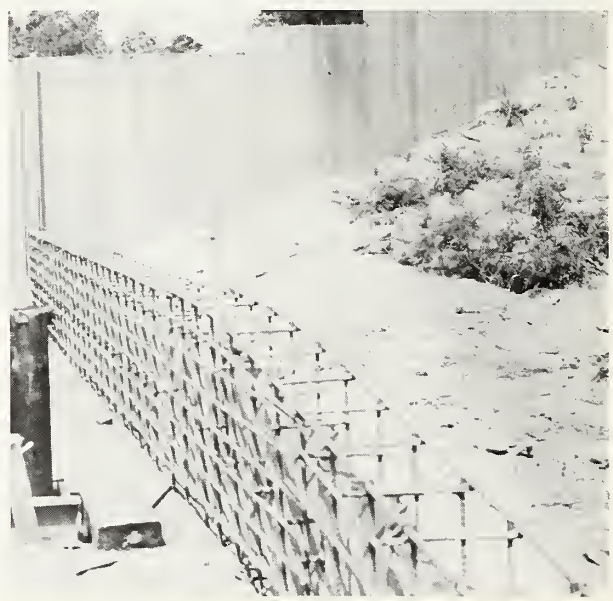

FIGURE 13.—Weir-crest reinforcing-steel arrangement.

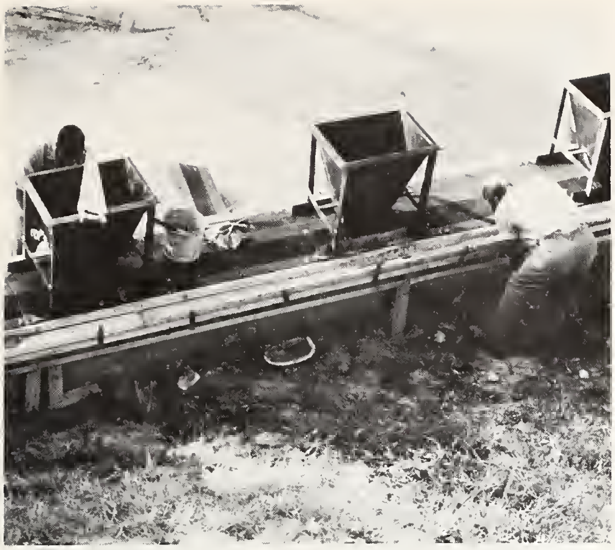

FIGURE 14.—Weir-crest forms and special funnel hoppers.

hoppers were constructed through which concrete could be poured into position. The concrete was designed for a 28-day strength of 4,500 lb/in².

The weir naturally serves as a dam, ponding all upstream water below the V-notch invert elevation. Because this water might need to be drained to facilitate maintenance, a 3-in-i.d. galvanized-steel pipe was formed through the weir wall directly below the V-notch invert. This pipe, placed 1 in above the apron floor, was threaded and capped at both ends. Caps normally remain in place but may be removed as desired for drainage (fig. 11).

All weirs, except station M, were designed and constructed with 10:1 side slopes in the V-notch section, which is centered in the measuring section. Station M was designed with 5:1 side slopes in the V-notch, which is in the left culvert barrel, and a level section in the right barrel. Throat depths, except at station M vary with size of the drainage area, ranging from 1.5 to 3.0 ft. Crests are level beyond the limits of the V-notch portion of the weir. V-notch invert elevations are 2 ft above the elevation of the stilling basins. Weir lengths and V-notch depths are given in table 2.

Stilling basin

Waterflow over weirs drops a vertical distance from 2 to 5 ft. This drop represents a considerable amount of energy that must be largely reduced to eliminate excessive down-

TABLE 2.—*Weir lengths and V-notch throat depths*

[Feet]

Gaging sites	Weir lengths	V-notch depths
B (main structure)	227.72	3.05
B (auxiliary structure)	75.20	(1)
F	141.39	2.00
I	87.28	1.63
J	55.13	1.55
K	58.37	1.45
M (V-notch) [2]	6.003	.63
M (horizontal)	5.958	(1)
N	48.71	2.04
O	48.60	2.05
UR	42.32	1.50

[1] Horizontal.
[2] V-notch portion in left barrel, horizontal portion in right barrel of box culvert.

stream scour. Since this could not only cause extreme soil loss below the structure but could also undermine the structure itself, a concrete apron, or stilling basin, was constructed downstream of the weir.

Aprons were of three basic designs. At box-culvert sites, the apron was merely a concrete slab extending from the weir downstream to the culvert floor. These aprons were level and at the same elevation as the upstream edge of culvert floors (fig. 11). Aprons were supported in the same manner as the large aprons at bridge sites. Concrete was designed for a 28-day strength of 3,000 lb/in².

In early phases of construction, the aprons at bridge sites extended 25 ft downstream from the weirs. These aprons were plain flat slabs with no energy dissipation devices added. Model studies discussed earlier resulted in an improved design that reduced costs and downstream-scour damage. Energy dissipation devices consisted of two components constructed on an apron extending only 10 ft downstream from the weir (fig. 15).

Baffle blocks consisting of 9-in cubes in two staggered rows were only used downstream of the V-notch portion of the weir. Rows are 9 in apart, and blocks are 27 in apart in the rows. At the downstream edge of the apron is a flipsill 6 in high extending across the entire apron width, with 3-ft openings at each end that provide apron drainage (*2*).

Support for the 12-in-thick reinforced-concrete apron was supplied by 8-in nominal

FIGURE 15.—Improved design of stilling basin, showing baffle-block arrangement and flip sill.

FIGURE 16.—Timber piles for support of concrete apron.

FIGURE 17.—Walls of dual-compartment stilling well.

diameter treated timber piles. The piles were spaced approximately 3.5 ft on center in the direction of flow and approximately 12 ft on center across the apron (fig. 16). Piles were driven approximately 15 ft deep.

Heavy riprap, placed downstream from aprons at bridge sites for a distance of 25 ft, served as an effective supplement to apron energy dissipators in reducing downstream scour (fig. 8). Riprap was not required at box-culvert sites except at station M.

Stilling-well-recorder shelters

Because both upstream and downstream water-surface elevations must be recorded to rate precisely the measuring structures during submerged conditions, two stilling wells and two stage recorders were used at each site. A monolithic dual-compartment stilling well with inside dimensions of 18 by 18 in was constructed at each site, supported by five treated timber piles similar to those supporting the concrete aprons. The walls of the concrete structure, 8 in thick, were designed for a 28-day strength of 4,500 lb/in^2 (fig. 17). Watertight steel doors were installed near the bottom of stilling wells to provide access for cleanout or other servicing (fig. 18). Similar service doors were also installed near the top of the stilling wells (fig. 17).

A 3-in-i.d. section of galvanized-steel pipe was installed through the wall of each well compartment, extending approximately 6 in outward from the wall (fig. 18). These were connected to intake pipes from the flow path.

Recorder shelters were constructed as an

FIGURE 18.—Lower section of stilling well, showing cleanout doors and downstream intake pipe with riser for flushing.

upward extension of the stillingwells (fig. 19). Shelter walls were constructed of 4-in-thick concrete. Ventilation of the shelters is provided by the insertion of one short section of 2-in-i.d. steel pipe through each side wall and two sections in the rear well. Pipes were screened to prevent entry of insects. An elbow turned downward was added to the outer end of each pipe to prevent rifle bullets from hitting the recorders (fig. 20).

In order to reduce the likelihood of damage to recorders by bullets, the doors are bulletproof. The doors were constructed of ¼-in steel plate over ¾-in plywood (fig. 19). Because the original design with the doors hinged from the top was found to be unsatisfactory, the doors were hinged from the side in later construction.

Stilling-well-recorder shelters were installed 10 ft outside weir guidewalls at approximately the same distance from the roadway as the weirs. A steel walkway with side rails (fig. 20) provided access from the road shoulder.

Intakes

As discussed earlier, both upstream and downstream water-surface elevations are needed to determine flow rates under submerged conditions. Therefore, both upstream and downstream flows were connected to their respective water-storage compartments of the stilling well.

Upstream intakes consisted of a 10-ft section of perforated 3-in pipe placed across the channel 10 ft upstream of the V-notch. The pipe

FIGURE 19.—Recorder shelter atop stilling wells, showing heavy duty door, digital stage recorders (one cover removed), and electronic timer.

FIGURE 20.—Streamflow-measuring facilities at station UR, showing stilling-well-recorder shelter with vent pipes, weir with 6-in angle iron crest attached, and walkway to recorder shelter.

was capped on the outer end and welded to two 8- by 8-in H-beams driven approximately 12 ft into the channel bed. Webs of the H-beams were cut out to cradle the pipe. The perforated pipe was installed at an elevation below V-notch invert elevation and at or above the elevation of the intake pipe into the stilling well, and it was connected to the stilling-well-intake pipe with standard 3-in-i.d. galvanized pipe. The connecting pipe was supported by intermediate H-piles driven into the bed as needed. At a point near the guidewalls a capped 3-in riser was installed for the flushing of sediment (fig. 11).

The original design of the downstream intakes consisted of two 6- by 6-in troughs or gutters in the apron. The first ran in the direction of the flow for most of the distance

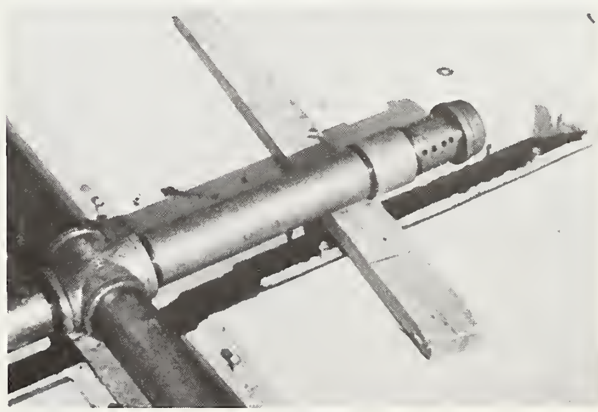

FIGURE 21.—Center portion of downstream intake-pipe assembly.

FIGURE 22.—Downstream intake covered with steel grating (original design).

from the V-notch towards the downstream edge of the apron, and the second ran across the apron, intersecting the first near the center. Three-inch-i.d. pipe with alternate 1-ft sections of perforated 3-in-i.d. pipe and 3-ft unperforated pipe were placed in the gutters. The two pipes were connected with a cross (fig. 21). This assembly was connected to the intake pipe into the downstream compartment of the stilling well. As with the upstream intake, a riser was installed near the guidewalls for possible flushing. After the gutters were backfilled with washed gravel, a steel grating was bolted over the gutter (fig. 22). The original design of the downstream intakes was abandoned in favor of a much simpler and more economical design developed as a result of hydraulic model tests.

FIGURE 23.—Downstream intake covered with steel grating (later design).

The improved design required only a 21- by 21-in intake well, 8 in deep, located in the apron corner formed by the downstream side of the weir and the guidewalls and placed on the apron end near the stilling well. The model tests revealed that this was a point of stagnation, which is a satisfactory point for obtaining downstream water-surface elevations, rather than the requirement that the integrated depth be determined across the entire apron, as called for by the earlier design.

A 12-in length of perforated 3-in-i.d. steel pipe was placed in this well and connected to the intake pipe into the appropriate compartment of the stilling well. The well was backfilled with washed gravel, and a steel grating was bolted in place over the well (fig. 23).

In all intake designs, the pipe was passed through the steel-piling guidewall. Care was taken to insure a proper seal around the pipe in order to prevent leakage, which would cause the recording of erroneous water-stage elevations.

Footbridges

Current-meter measurements at the gaging stations were necessary to check model ratings and to establish stage-discharge relations. Measurements can be made while wading as long as depths and velocities permit. The need for a safe flow-measurement site at high discharge was met by the construction of a footbridge, or catwalk, at each gaging site.

FIGURE 24.—Footbridge at highway bridge with dolly and equipment for flow measurement.

Footbridges were attached to the upstream side of highway bridges (fig. 24) and to the downstream side of box culverts (fig. 25). Footbridge supports of 6-in by 8.2-lb steel channel at box culverts and 8-in by 13.75-lb channel at bridges were anchored with lag bolts and braced with angle sections to provide required strength and rigidity. Runners were of the same material as supports. Angle sections of 1½ by 1½ by ¼ in were used to construct standards and braces for bridge side rails. Footbridges are 30 in wide on the inside and 42 in high.

Atop the standards are 4- by 3- by ¼-in angles for horizontal rails. These rails, in addition to simply being part of the footbridges, also serve as the tracks for a current-meter dolly. As shown in figure 24, this dolly accommodates a reel used in streamflow measurement and facilitates measurements at any intermediate point. Distances across the section are permanently marked on the top rail, insuring accuracy of flow width measurements.

The first footbridges were constructed with floors of 2- by 6-in treated lumber, but this design was later changed, and expanded steel gratings were used. The steel is much simpler to install and requires less maintenance.

Station M

Gaging site M, which drains 1.20 mi^2 in the headwaters of Newell Branch (fig. 2), is an exception to some of the instrumentation and construction described for other sites. At this site the weir was built within the downstream end of the box culvert. Because there was adequate overfall from the culvert to prevent submergence, only the upstream stage recorder was necessary. A small V-notch weir with 1:5 side slopes was constructed in one culvert barrel. A horizontal crest was installed in the other barrel at an elevation above the upper limits of the V-notch portion. No stilling basin was constructed below the culvert, but riprap was used to prevent downstream scour. Other aspects of installation are the same as at other sites (fig. 10).

FIGURE 25.—Footbridge at box culvert.

Recorders

Upstream and downstream water-surface elevations are recorded in increments of 0.01 ft by two Fischer-Porter digital stage recorders (fig. 19). The recorders and their operation are discussed in reference *3*.

RESULTS

Operation and Performance

The performance of all streamflow-measuring installations and stage recorders has been good. With little exception, all weirs have performed as designed and constructed. Few data were available at most sites to determine the necessary weir-crest elevation. Visual observations of water marks and information received from area residents were utilized in determining the probable required elevations. Since the weir crests could be raised much more easily and less expensively than lowering them, caution was exercised in this decision. As a result, it became necessary to raise weir-crest elevations 0.5 ft at three sites (I, K, O) in LRW after about 1 year of operation. This was also done at site UR, the adjacent watershed under study to determine the effects of urbanization on watershed hydrology. Temporary increases in the weir-crest elevations were obtained by attaching a 4- by 6-in steel angle as shown in figure 20. This steel angle was replaced by concrete after a determination that a 0.5-ft increase was satisfactory. No other problems have been encountered in the performance of the facilities as designed. Downstream scour has been minimal with both types of aprons at highway bridge sites. Recorders have performed as anticipated, with a minimum of lost records.

Data

From the beginning of the installation of the streamflow-measuring facilities in 1967 until

completion of station B in October 1971, major efforts were expended in planning and implementing the research program, instrumentation, and construction of necessary facilities. Since the installation was completed, the handling, processing, and analysis of data have received major emphasis. Computer programs have now been developed to transfer streamflow data that had been stored on computer cards onto magnetic tape. All streamflow data through 1972 have now been placed on magnetic tape. Initial stage-discharge relations based on streamflow measurements have been developed for all gaging sites. Data processing is continuing.

A discussion of data processing is not within the scope of this report: references *3* and *12* contain discussions of data collection, handling, and processing.

COMMENTS AND RECOMMENDATIONS

Although construction in LRW generally went smoothly and according to schedule, a number of seemingly minor items could have caused a delay in the construction, increasing the cost of the project. Proper planning and scheduling was imperative. Because construction was entirely within and adjacent to the stream channel, work had to be scheduled for the dry season. Contracts were arranged to make the work schedule flexible, allowing work to begin as soon as weather conditions permitted.

Cost-plus contracts allowed for required modifications in design or construction to be readily made on the job with minimum delay. Weather conditions could greatly alter work schedules. Flexibility of the cost-plus contracts permitted such changes to be easily managed. The Government construction superintendent maintained control of scheduling operations. Good cooperation and understanding between the construction superintendents was a necessity.

Work was normally scheduled for 5 days a week to eliminate overtime pay. However, the Government superintendent was authorized to schedule Saturday work when it seemed advisable. An example of a wise decision to work on Saturday occurred at station F. The placement of forms and reinforcing steel for the concrete-apron stilling basins was completed late Friday afternoon. He directed that concrete be poured on Saturday rather than waiting until Monday. A heavy thunderstorm sent approximately a 3-ft depth of water over the newly placed concrete Sunday afternoon. Had the concrete not been in place, all the steel would have required removal, cleaning, and replacing. Also, much silt would have had to be removed by hand and the apron base reshaped by hand. A sizable extra cost was avoided in this case by the judicious scheduling of one day's overtime.

Proper design, planning, and cooperation between the Government and contractors allowed all construction in LRW, except at station B, to be completed with almost no time loss. Construction of station B, the outlet for the 126.4-mi^2 drainage area, was delayed because of limited funds and weather conditions. Although construction began in 1969, funds were not available for completion. Portions that would not be damaged by high winter flows were installed in 1969 with plans to complete construction the following year. However, heavy rains and high flow rates after the weir was constructed in 1970 prevented the installation of the downstream apron. Winter and spring high flows damaged the center section of the constructed weir, causing extensive scour beneath the weir and the support wall. The center section buckled, with the concrete breaking. This section of the weir (without the steel-piling cutoff wall) had to be removed in 1971. The steel piling was pulled back into position, and the weir crest replaced. A large amount of rock riprap and concrete was then placed around the piling to hold it in place. Construction was then completed satisfactorily with no additional problems. Performance of station B has been most satisfactory, as it has been at all other sites.

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*-1976-G P 0-1750S/771-040/1

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